Vacuum and surcharge combined one-dimensional consolidation of clay soils

E. Mohamedelhassan and J.Q. Shang

Abstract: In this study, a vacuum and surcharge combined one-dimensional consolidation model is developed. Terzaghi's consolidation theory is revisited by applying the initial and boundary conditions corresponding to combined vacuum and surcharge loading on a soil. A test apparatus is designed, manufactured, and assembled to verify the model. The apparatus has the capacity of applying designated vacuum and surcharge pressures to a soil specimen, and it allows for the measurement of the excess pore-water pressure, settlement, and volume change during the consolidation process. Two series of tests are performed using the apparatus on two reconstituted natural clay soils, namely, the Welland sediment at water contents close to its liquid limit and the Orleans clay, reconstituted and consolidated under an effective stress of 60 kPa. The former test series mimics the strengthening of a very soft soil, such as the hydraulic fill used in land reclamation. The latter test series is designed to study vacuum–surcharge combined strengthening of a consolidated soil. It is demonstrated from the experiments that the one-dimensional vacuum–surcharge consolidation model describes the consolidation behaviour of both soils well. The consolidation characteristics of the soils show no discrimination against the nature of the consolidation pressure, namely, whether they are consolidated under the vacuum pressure alone, under the surcharge pressure alone, or under a pressure generated by the combined application of vacuum and surcharge. The study concluded that the soil consolidation characteristics obtained from the conventional consolidation tests can be used in the design of vacuum preloading systems, provided that the one-dimensional loading condition prevails.

Key words: consolidation, soil improvement, vacuum pressure, surcharge pressure, excess pore-water pressure, soil consolidation parameters.

Résumé : Dans cette étude, on développe un modèle de consolidation unidimensionnelle par l'application combinée du vide et d'une surcharge. La théorie de consolidation de Terzaghi est révisée en appliquant les conditions aux frontières et initiales correspondant à une combinaison de chargement par vide et par surcharge. Un appareil d'essai est conçu, fabriqué et assemblé pour vérifier le modèle. L'appareil peut appliquer à un spécimen de sol des pressions de vide et de surcharge choisies, et permet la mesure de l'excédent de pression interstitielle, du tassement et du changement de volume durant le processus de consolidation. Deux séries d'essais ont été réalisés au moyen de l'appareil sur deux sols argileux naturels reconstitués, nommément le sédiment de Welland à des teneurs en eau près de la limite de liquidité, et l'argile d'Orléans, reconstitués et consolidés sous une pression effective de 60 kPa. La première série d"essais simule le renforcement d'un sol très mou tel qu'un remblai hydraulique utilisé dans la réhabilitation des terrains. La deuxième série d'essais est conçue pour étudier le renforcement d'un sol consolidé par la combinaison du vide et de la surcharge. En partant de ces expériences, on démontre que le modèle de consolidation unidimensionnelle par vide-surcharge décrit bien le comportement de consolidation des sols. Les caractéristiques de consolidation des sols ne montrent aucune distinction en fonction de la nature de la pression de consolidation, nommément, qu'elle soit générée par le vide seulement, par une surcharge, ou par une combinaison de vide et de surcharge. L'étude à conclu que les caractéristiques de consolidation obtenues par des essais conventionnels de consolidation peuvent être utilisées dans la conception de systèmes de préchargement sous vide, pourvu que la condition unidimensionnelle de chargement prévale.

Mots clés : amélioration du sol, pression de vide, pression de surcharge, excédent de pression interstitielle, paramètres de consolidation du sol.

[Traduit par la Rédaction]

Introduction

Vacuum preloading is a technique used to improve the strength of soft clayey soils. In vacuum preloading applications, the surface of the ground is sealed with a membrane and through a vacuum pump a negative pressure is created in the sand cushion beneath the sealing membrane and in the vertical drains installed in the soil. The change in the soil

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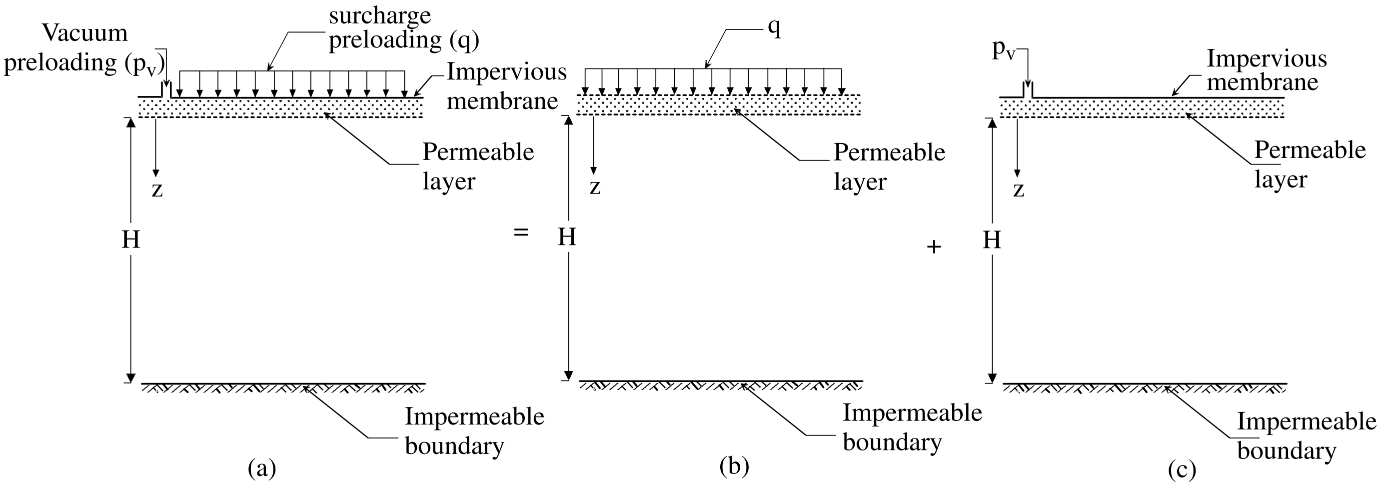
Fig. 1. Schematic of (*a*) vacuum and surcharge combined preloading; (*b*) surcharge preloading; and (*c*) vacuum preloading.



pore-water pressure produced from the applied vacuum pressure induces discharge of the soil water and consolidation thereby increasing the soil shear strength.

Kjellman (1952) first introduced the concept of using vacuum preloading to improve the soil strength. One of the first applications of the technique was in 1957 when it was used in a runway extension at Philadelphia International Airport (Halton et al. 1965). In recent years, many successful field applications have been reported in the literature, including the use of vacuum preloading in a land reclamation project in China (Shang et al. 1998). Tang and Shang (2000) demonstrated the effectiveness of vacuum preloading consolidation in eliminating excessive settlement under static and dynamic loads on an airport runway. Chu et al. (2000) presented a successful case study whereby vacuum preloading was used to improve the soil strength at an oil storage station. Other vacuum preloading applications include the management of mine tailings (Shang and Zhang 1999).

To date, however, the design of vacuum preloading systems is primarily empirical. To fully understand the parameters involved in the vacuum preloading process and to optimize and predict treatment results, a systematic study is needed. An apparatus was designed and manufactured for the parametric study of one-dimensional vacuum consolidation. The objectives of this study are as follows:

(1) To establish a one-dimensional consolidation model that accommodates vacuum consolidation and vacuum and surcharge combined consolidation.

(2) To study the time-dependent excess pore-water pressure and deformation behaviour of soft clayey soils under a vacuum pressure using the newly developed vacuum consolidation apparatus and to verify the soil parameters under vacuum and surcharge preloading conditions.

(3) To compare the consolidation behaviour of clayey soils under vacuum preloading, surcharge preloading, and vacuum and surcharge combined preloading.

(4) To examine the soil consolidation parameters such as the coefficient of consolidation (c_v), coefficient of volume change (m_v), coefficient of compressibility (a_v), compression index (C_c), and hydraulic conductivity (k) from vacuum, surcharge, and vacuum and surcharge combined preloading tests.

(5) To examine the feasibility of using the soil parameters obtained from the conventional consolidation tests in the design and performance prediction of vacuum preloading consolidation applications.

Two clayey soils were used in this study. The first soil (Welland River sediment) was used to compose reconstituted samples and tested without prior consolidation, while the second soil (Orleans clay) was reconstituted and consolidated first and then tested.

Governing equation of vacuum and vacuum and surcharge combined consolidation

Figures 1*a*–1*c* present the schematics of vacuum and surcharge combined preloading, surcharge preloading, and vacuum preloading, respectively, in one-dimensional consolidation. The excess pore-water pressure generated by a vacuum and surcharge combined preloading, shown in Fig. 1*a*, may be evaluated by the law of superposition as the sum of the excess pore-water pressure generated by the surcharge preloading shown in Fig. 1*b* and the excess pore-water pressure induced by the vacuum preloading shown in Fig. 1*c*. The general differential equations for the excess pore-water pressure, u, for a surcharge preloading, q, and a vacuum preloading, p_v, are

$$[1a]\quad \frac{\partial u}{\partial t} = c_{vs}\frac{\partial^2 u}{\partial z^2} \qquad (0 < z < H,\ t > 0)$$

$$[1b]\quad \frac{\partial u}{\partial t} = c_{vv}\frac{\partial^2 u}{\partial z^2} \qquad (0 < z < H,\ t > 0)$$

where t is the time, z is the depth, H is the drainage path, c_{vs} is the coefficient of consolidation for surcharge preloading, and c_{vv} is the coefficient of consolidation for vacuum preloading. The boundary and initial conditions for eq. [1*a*], surcharge preloading, are:

$$[2]\quad u(0, t) = 0 \qquad (t > 0)$$, free drainage at the soil upper layer

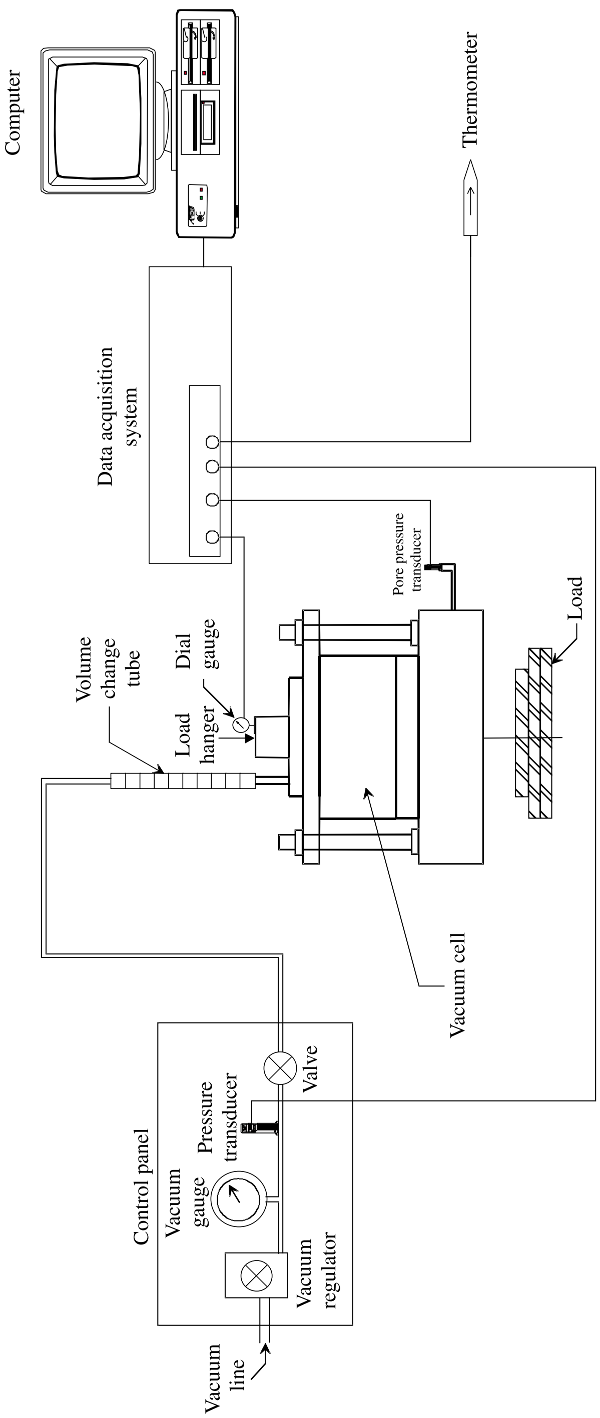


Fig. 2. Schematic of vacuum consolidation testing apparatus.

[3] $\dfrac{\partial u}{\partial z}(H,\ t) = 0 \qquad (t > 0)$, impervious boundary at the bottom of the soil layer

[4] $u(z,\ 0) = q \qquad (0 < z < H)$

In the surcharge consolidation case, the excess pore-water pressure is positive and ultimately reduces to zero. For eq. [1*b*], vacuum preloading, the boundary and initial conditions are:

[5] $u(0,\ t) = p_{\mathrm{v}} \qquad (t > 0)$, a constant vacuum pressure, p_{v}, at the soil upper layer

[6] $\dfrac{\partial u}{\partial z}(H,\ t) = 0 \qquad (t > 0)$, impervious boundary at the bottom of the soil layer

[7] $u(z,\ 0) = 0 \qquad (0 < z < H)$

In the consolidation by vacuum pressure the excess pore-water pressure is negative and ultimately approaches the applied vacuum pressure.

The assumptions used to derive the solution for the vacuum consolidation case are similar to those of Terzaghi's one-dimensional surcharge consolidation theory, which is a small strain theory. For the defined boundary and initial conditions the analytical solution of eq. [1*b*] for the vacuum preloading case is

$$[8] \qquad u_{\mathrm{v}}(z,\ t) = p_{\mathrm{v}}\left\{1 - \sum_{n=0}^{\infty}\frac{4}{(2n+1)\pi}\sin\left(\frac{2n+1}{2}\pi\frac{z}{H}\right) \times \exp\left[-\left(\frac{2n+1}{2}\right)^2\pi^2\frac{c_{\mathrm{vv}}t}{H^2}\right]\right\}$$

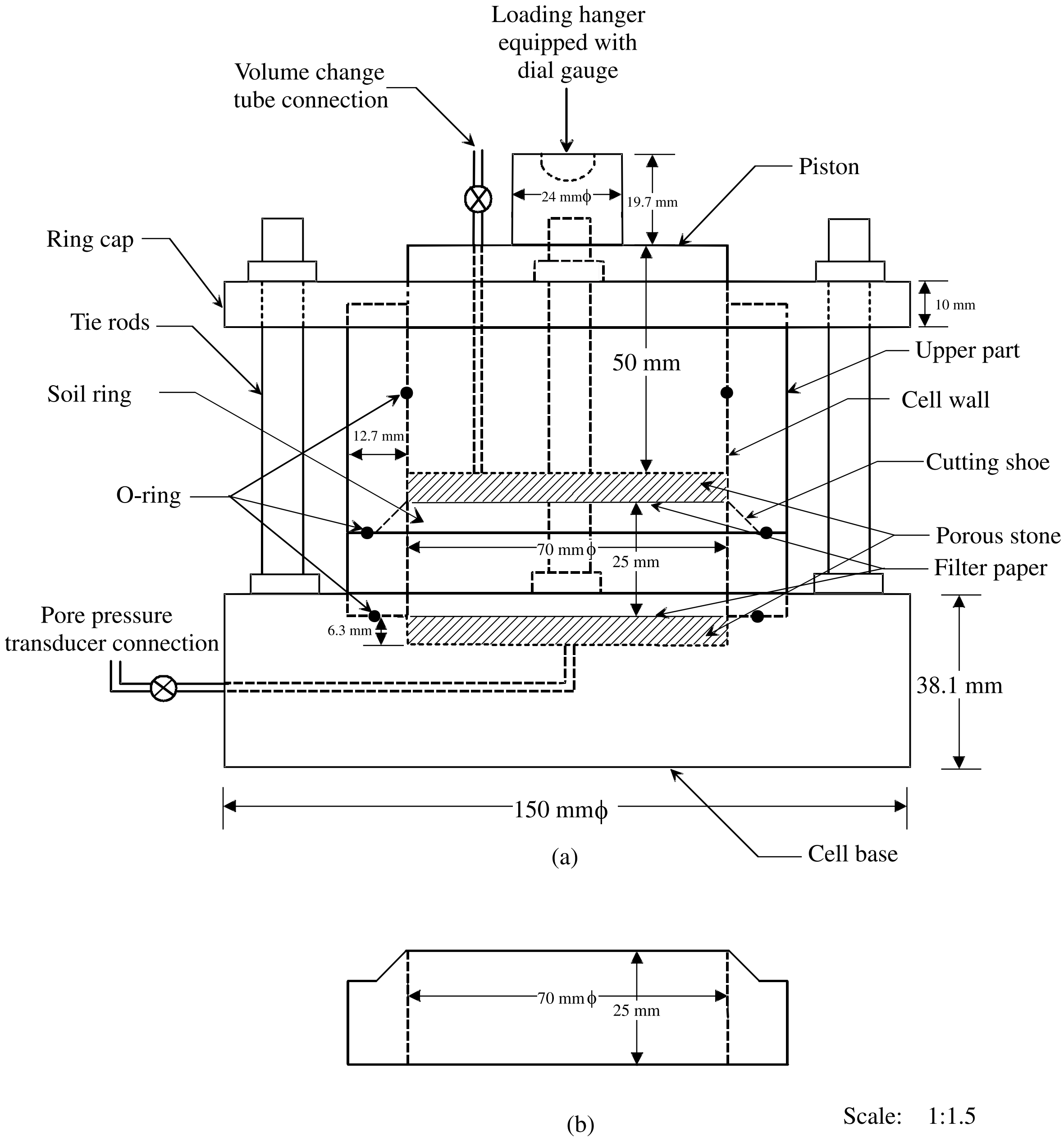
whereas for the case of surcharge preloading, eq. [1*a*], the solution is based on the well known Terzaghi's theory, i.e.

$$[9] \qquad u_{\mathrm{s}}(z,\ t) = q\sum_{n=0}^{\infty}\frac{4}{(2n+1)\pi}\sin\left(\frac{2n+1}{2}\pi\frac{z}{H}\right) \times \exp\left[-\left(\frac{2n+1}{2}\right)^2\pi^2\frac{c_{\mathrm{vs}}t}{H^2}\right]$$

In the case of vacuum and surcharge combined preloading consolidation, the excess pore-water pressure is the summation of eqs. [8] and [9]. If the coefficient of consolidation c_{vc} is assumed to represent the vacuum and surcharge combined preloading case, the summation of eqs. [8] and [9] yields

$$[10] \qquad u(z,\ t) = u_{\mathrm{v}}(z,\ t) + u_{\mathrm{s}}(z,\ t) = p_{\mathrm{v}} + (q - p_{\mathrm{v}})\sum_{n=0}^{\infty}\frac{4}{(2n+1)\pi}\sin\left(\frac{2n+1}{2}\pi\frac{z}{H}\right) \times \exp\left[-\left(\frac{2n+1}{2}\right)^2\pi^2\frac{c_{\mathrm{vc}}t}{H^2}\right]$$

Fig. 3. (*a*) Vacuum cell. (*b*) Soil ring, 70 mm ϕ inner diameter.



Alternatively, the general differential equation for the vacuum and surcharge combined preloading is

$$[11]\qquad \frac{\partial u}{\partial t} = c_{vc}\frac{\partial^2 u}{\partial z^2} \qquad (0 < z < H,\ t > 0)$$

and the boundary and initial conditions are

$$[12]\qquad \frac{\partial u}{\partial z}(H, t) = 0 \qquad (t > 0)$$, impervious boundary at bottom of soil layer

$$[13]\qquad u(0, t) = p_v \qquad (t > 0)$$, constant vacuum pressure, p_v, at the soil upper layer

$$[14]\qquad u(z, 0) = q \qquad (0 < z < H)$$

The solution of eq. [11] is given by

$$[15]\qquad u(z,t) = p_v + \sum_{n=0}^{\infty} A_n \sin\left(\frac{2n+1}{2}\pi\frac{z}{H}\right) \times \exp\left[-\left(\frac{2n+1}{2}\right)^2 \pi^2 \frac{c_{vc}\, t}{H^2}\right]$$

where

$$[16]\qquad A_n = \frac{2}{H}\int_0^H q \sin\left(\frac{2n+1}{2}\pi\frac{z}{H}\right)dz - \frac{4}{\pi^2}\frac{(2n+1)\pi\, p_v}{(2n+1)^2} = \frac{4(q-p_v)}{(2n+1)\pi}$$

Substituting A_n from eq. [16] into eq. [15] yields eq. [10], i.e., both alternatives give identical solutions.

The average degree of consolidation for the vacuum and surcharge combined preloading can be expressed as

$$[17] \quad U_{ave} = \frac{\int_0^H (u_i - u)}{\int_0^H (u_i - u_f)} = \frac{\int_0^H (q - u)\,dz}{\int_0^H (q - p_v)\,dz} = 1 - \sum_{n=0}^{\infty} \frac{8}{(2n+1)^2 \pi^2} \exp\left[-\frac{(2n+1)^2}{4} \pi^2 \frac{c_{vc}\, t}{H^2}\right]$$

where u_i and u_f are the initial and final excess pore-water pressures, respectively. As seen in eq. [17], the average degree of consolidation, U_{ave}, for the vacuum and surcharge combined preloading has the same form as that proposed by Terzaghi.

If a soil under vacuum consolidation or vacuum and surcharge combined consolidation demonstrates the same behaviour as that of a surcharge pressure of the same magnitude, the design of vacuum consolidation systems can be based on the conventional consolidation test results. However, to do so, one must first examine the consolidation behaviour of soil under both vacuum preloading pressure and vacuum–surcharge combined preloading pressures.

Vacuum consolidation testing apparatus

The design of the vacuum preloading testing apparatus considered the following aspects:

(1) It should be able to generate a vacuum pressure, a surcharge pressure, or a simultaneously combined vacuum and surcharge pressure in a soil sample at predetermined magnitudes. The ability to apply a vacuum pressure with or without a surcharge pressure to the soil sample being tested is a primary criterion considered in the design of the apparatus. This requires that the test be conducted in an airtight pressurized system. In the meantime, the system should allow the soil sample to deform with minimum frictional resistance.

(2) It should be able to monitor the soil pore-water pressure, settlement, and volume change during the test at predetermined time intervals. With respect to this design consideration, the monitoring is of particular importance in understanding the mechanism of the vacuum preloading process.

The schematic of the vacuum testing apparatus is shown in Fig. 2. The soil sample is placed in the vacuum cell situated on a loading frame that allows the application of a vertical levered surcharge load through a hanger to the soil sample. A vacuum pressure line capable of applying a maximum vacuum pressure of ~80 kPa is used to provide the vacuum pressure. The vacuum line is connected to a vacuum regulator with a vacuum gauge and a pressure transducer. This allows adjustment to the desired vacuum pressure and ensures a stable vacuum pressure during the test period. The regulated vacuum pressure is connected with a valve to the volume change tube, which in turn is connected to the upper part of the vacuum cell. Thus, the regulated vacuum pressure is applied to the top of the soil specimen. The vacuum regulator, vacuum gauge, pressure transducer, and the valve were mounted on a control panel, as shown in Fig. 2. The volume change tube consists of a burette 60 cm long and 0.95 cm in inner diameter. The applied vacuum pressure is monitored and recorded throughout the testing period via the pressure transducers connected to a data acquisition system and a computer. A second pressure transducer is connected to the base of the vacuum cell to measure the changes in pore-water pressure with time at the bottom of the soil sample. The readings of the transducer are also recorded by the data acquisition system. An electronic dial gauge mounted on the loading piston of the vacuum cell is connected to the data acquisition system to monitor and record the soil settlement with time. A thermometer connected to the acquisition system is used to monitor the room temperature during the test period.

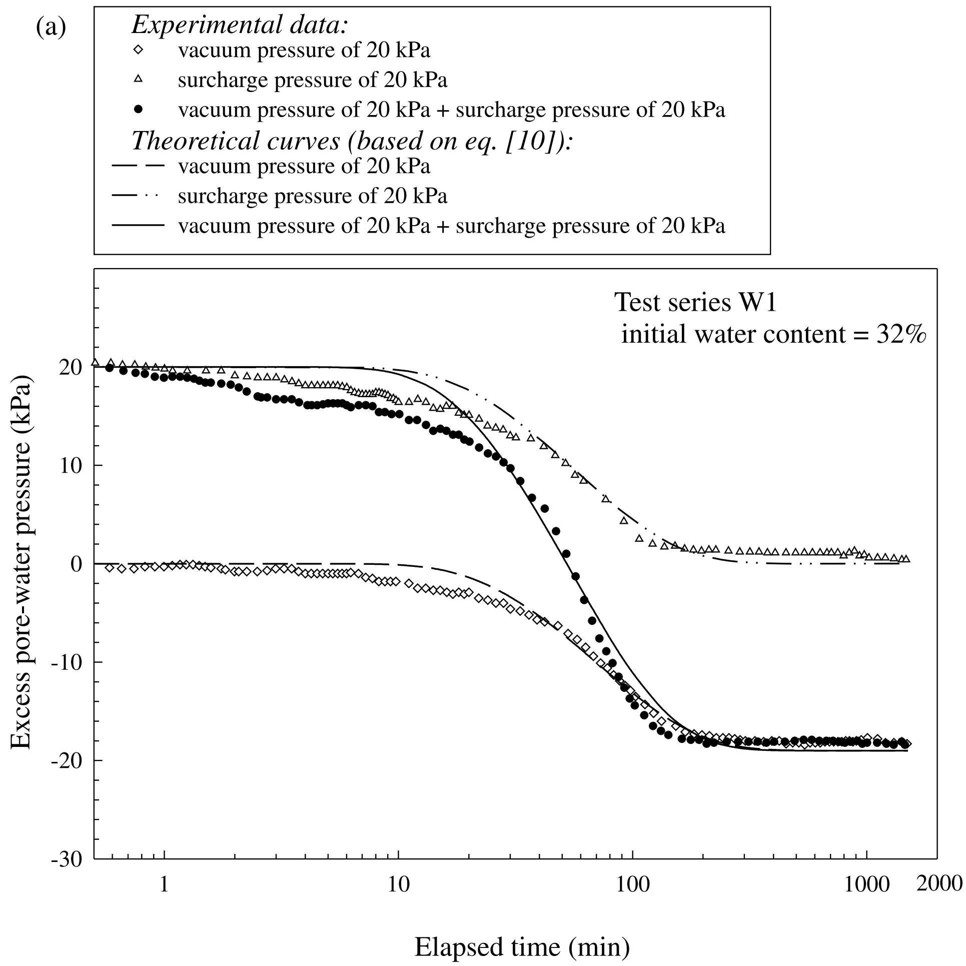
The vacuum cell, shown in Fig. 3*a*, was made of stainless steel. There are two drainage channels that are used to apply the vacuum preloading pressure and measure the pore-water pressure. The drainage channel through the top cap (piston) is used to apply the vacuum pressure and the channel through the base to monitor the pore-water pressure at the bottom of the sample. The cell can accommodate a soil sample 70 mm in diameter and 25 mm in height. These dimensions were chosen as similar to the dimensions of the standard oedometer test. The stainless steel ring (soil ring) used to hold the soil sample was designed with a sharp cutting shoe as shown in Fig. 3*b* to enable testing of undisturbed soil samples.

One of the challenges in the design of the vacuum cell was to ensure an airtight seal and yet allow a negligible friction between the piston and the wall of the cell. The airtight seal was achieved by using an O-ring as show in Fig. 3*a*, whereas a very smoothly finished piston and inner cell wall lubricated with silicon grease significantly reduced the friction. In the loading calibration trials of the cell, over 96% of the load applied on the levered arm transferred to the soil sample, as indicated by the pressure transducer readings at the bottom of the sample.

Table 1. Summary of properties of Welland River sediment.

Sediment properties	
Liquid limit, %	31.5
Plastic limit, %	19.9
Plasticity index, %	11.6
Specific gravity	2.73
Cation exchange capacity, mequiv./100 g of soil	11
Sand-sized, %	12
Silt-sized, %	60
Clay-sized, %	28
Mineralogy	
Chlorite, %	~18.3
Illite, %	~14.7
Quartz, %	~40
Feldspar, %	~4
Calcite, %	~14
Dolomite, %	~9

Fig. 4(*a*). Excess pore-water pressure versus elapsed time for test series W1.



The soil sample was placed in the soil ring between the two porous stones shown in Fig. 3*a*. A filter paper was placed between the soil sample and the porous stone to prevent clogging of the stones by fines during testing.

Experimental program

Four sets of tests were conducted to examine the vacuum and surcharge combined one-dimensional consolidation model and evaluate the soil parameters during the consolidation process. Reconstituted soil samples were used in the first three sets of tests (W1 to W3), whereas reconstituted–consolidated samples were used in the fourth set of tests (O1).

Sample preparation

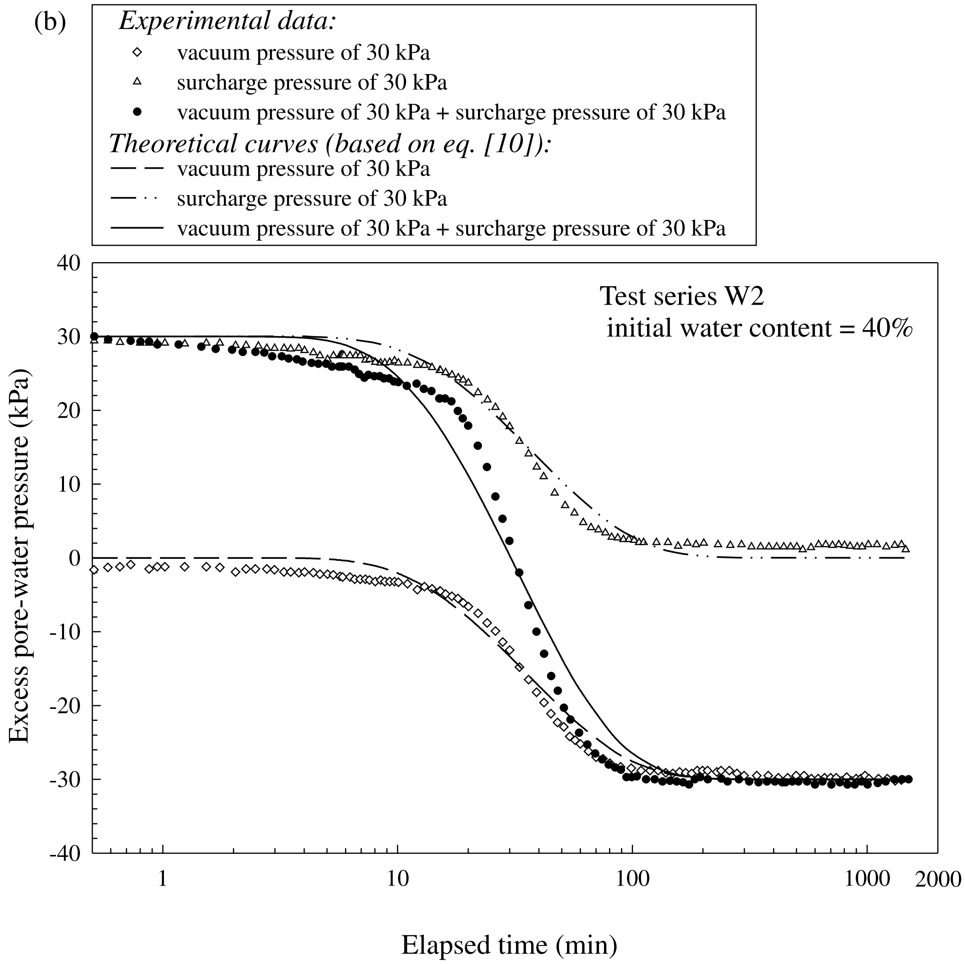
The vacuum cell was designed to test reconstituted and undisturbed soil samples. The vacuum cell's performance was first examined using a reconstituted soil following Burland's (1990) method. The reconstituted soil specimen was prepared with the water content between 1 and 1.5 times the liquid limit. The compressibility of the reconstituted soil represents the intrinsic property of the soil under this condition and is independent of its natural state. The reconstituted sample was carefully placed in small patches in the soil ring and gently compacted with a thin rod to remove any noticeable air voids. However, it is quite possible that soil samples prepared by this method may not be fully saturated at the beginning of testing.

The soil specimen used for the reconstituted–consolidated samples was carefully trimmed to a diameter slightly larger than 70 mm and a height of approximately 30 mm. The sample specimen was then laid on a smooth flat surface and the soil ring, shown in Fig. 3*b*, was pushed gently via its cutting shoe edge against the sample. The soil in excess of the soil ring was then carefully cut and removed from the ring edges.

Testing procedure

The two porous stones, shown in Fig. 3*a*, were boiled in distilled water until saturated. The lines in the base of the cell were examined and saturated with water and the pressure transducers were calibrated following the same procedure as the standard triaxial test. A saturated porous stone was placed at the base of the cell followed by a wet filter paper. The soil ring was then placed on the filter paper followed by another wet filter paper and the second porous stone. The upper part of the cell, shown in Fig. 3*a*, was then put in place. The piston was lubricated and pushed carefully into the upper part of the cell. As the O-ring passed into the upper part, the piston slid gently on its own weight and rested at the upper porous stone. A ring cap, shown in Fig. 3*a*, was placed on top of the cell and tightened using

Fig. 4(*b*). Excess pore-water pressure versus elapsed time for test series W2.



three tie rods to hold the cell together. The line between the upper porous stone and the volume change tube was saturated with water. The cell was then placed on the loading frame and connected to the vacuum line with the valve turned off. The settlement gauge, the two pressure transducers, and the thermometer were connected to the data acquisition system, which was connected to a computer. After adjusting the vacuum preloading pressure to the desired value, the valve between the vacuum line and the volume change tube was opened. The water expelled from the soil sample was monitored during the test period through the volume change tube and recorded as a backup check. The loading frame allowed for the application of a surcharge pressure prior to or at any time during the vacuum testing period. The test was continued until the primary consolidation was completed. This is normally determined through the plots of the excess pore-water pressure and settlement versus elapsed time. During all of the tests discussed in the paper, the room temperature was monitored and recorded and the fluctuations were found to be less than 1°C.

Results and discussion

After the vacuum apparatus was built, assembled, and calibrated; reconstituted and reconstituted–consolidated soil samples were tested to evaluate the performance of the apparatus, verify the one-dimensional vacuum and surcharge combined consolidation model, and study the vacuum preloading effects on soft clayey soils in terms of the excess pore-water pressure and soil consolidation parameters. The results of the four series of tests conducted in this study are summarized in Table 3 and presented and discussed in the following sections.

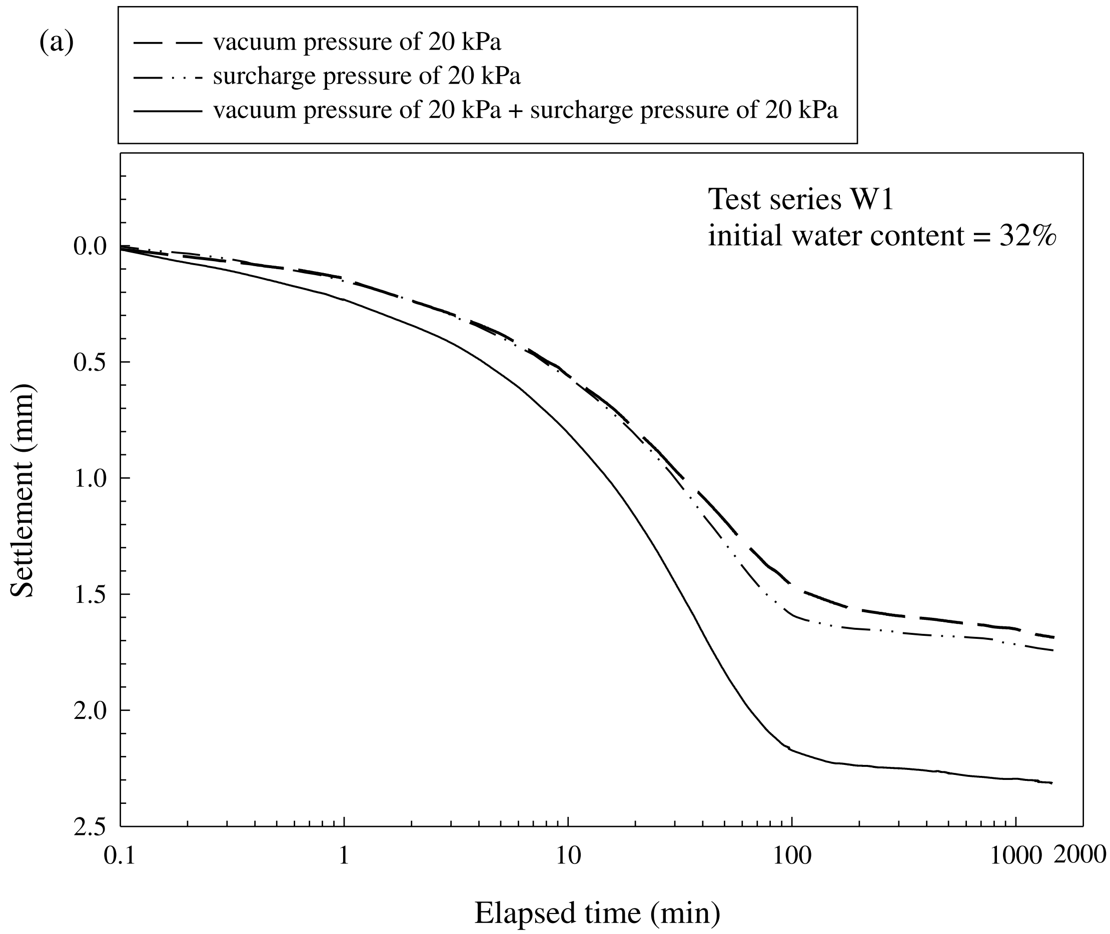
Reconstituted soil sample

The reconstituted sample was prepared from the Welland River (Ontario, Canada) sediment. The sediment was dredged from the bottom of the river and its soil composition and properties are shown in Table 1. The sediment is classified as a clayey silt with 12% sand, 60% silt, and 28% clay. The Atterberg limits tests indicated that the material has a low plasticity. X-ray diffractions and chemical analysis were conducted to determine the mineral composition of the sediment.

Excess pore-water pressure and settlement

Two test series, W1 and W2, were performed on the Welland River sediment to study the excess pore-water pressure and settlement with vacuum pressure. The water contents of test series W1 and test series W2 were 32% (approximately equal to the liquid limit) and 40% (approximately 1.25 times the liquid limit), respectively. The water

Fig. 5(*a*). Settlement versus elapsed time for test series W1.



contents were chosen according to the procedure described by Burland (1990). Three identical soil specimens were prepared at each of the two water contents. Sample W1.1 was tested under a vacuum pressure of 20 kPa and sample W1.2 was tested under a surcharge pressure of 20 kPa. Sample W1.3 was tested at a combined vacuum and surcharge pressures of 20 kPa each. Samples W2.1, W2.2, and W2.3 were tested at 30 kPa vacuum pressure, 30 kPa surcharge pressure, and 60 kPa, consisted of 30 kPa vacuum pressure and 30 kPa surcharge pressure, respectively.

Figures 4*a* and 4*b* present the measured excess pore-water pressure versus elapsed time for test series W1 (tested at a water content of 32%) and test series W2 (tested at a water content of 40%). The theoretical predictions from eq. [10] are also plotted in the figures. It is clearly shown that (1) surcharge loading generates a positive excess pore-water pressure that dissipates to zero with time; (2) vacuum preloading generates a negative pore pressure that approaches the applied pressure with time; and (3) a combined surcharge and vacuum preloading is a superposition of (1) and (2) above, i.e., the consolidation starts with an excess pore-water pressure equivalent to the surcharge pressure and ends with a negative pore pressure equivalent to the applied vacuum pressure.

As seen in Figs. 4*a* and 4*b*, the experimental results agree well with the theoretical prediction of eq. [10]. Larger deviations between the model and experimental data are observed in the first 10–20 min of the tests. This may be attributable mainly to two factors, i.e., the reconstituted soil specimens were not fully saturated at the beginning of the tests and the assumption of small strain in the development of Terzaghi's consolidation theory.

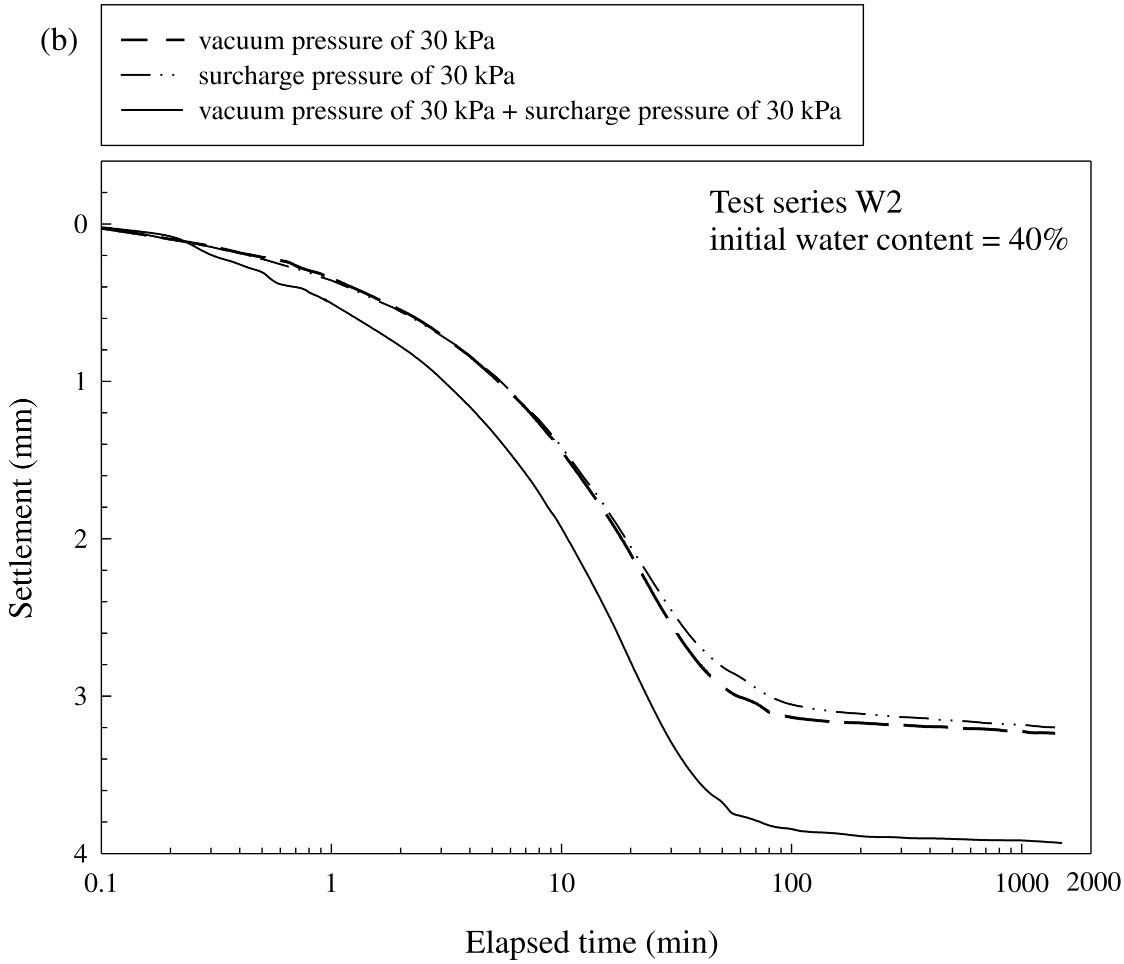
Figures 5*a* and 5*b* present the soil settlement versus elapsed time for test series W1 and W2, respectively. The results further indicate that the nature of the consolidation pressure, either surcharge or vacuum, has no bearing on soil consolidation.

Soil consolidation parameters

For a quantitative evaluation, Table 3 presents the coefficients of consolidation (c_v) for test series W1 and W2, obtained by the best fit of the experimental data using the computer software Mathcad (MathSoft Inc. 2000). It may be seen that for the same magnitude of consolidation pressure, the c_v values are nearly identical for vacuum and surcharge consolidation (tests W1.1 and W1.2, tests W2.1 and W2.2).

The soil consolidation parameters were further investigated in test series W3. In this series, three soil samples were reconstituted at a water content of 32%. The first sample, W3.1, was consolidated at vacuum pressures of 15, 30, 45, and 60 kPa and unloaded to 15 kPa. The second sample, W3.2, was consolidated with surcharge pressures of 15, 30, 45, and 60 kPa and unloaded to 15 kPa. The soil sample W3.3 was consolidated with a vacuum and surcharge

Fig. 5(*b*). Settlement versus elapsed time for test series W2.



combined pressure used in the previous two samples, i.e., 15 kPa vacuum + 15 kPa surcharge, 30 kPa vacuum + 30 kPa surcharge, etc. The results are summarized in Table 3. Figure 6 shows the applied consolidation pressure versus the void ratio for test series W3. The soil demonstrated classic virgin compression behaviour during loading with virgin compression index C_r = 0.13 and a recompression index C_r = 0.015. It is clearly shown in the figure that the nature of the applied consolidation pressure (vacuum, surcharge, or both combined) has no bearing on the compression behaviour of the soil.

Figure 7 shows the coefficient of consolidation, c_v, versus the applied pressure for test series W3. As clearly illustrated in the figure, the vacuum and surcharge preloading of the same magnitude produces similar coefficients of consolidation. The figure also shows that the coefficient of consolidation obtained by a vacuum and surcharge combined preloading is similar to the one obtained by a vacuum or surcharge pressure of the combined magnitude.

Figure 8 shows the hydraulic conductivity, *k*, of the Welland sediment versus the average void ratio, e_{ave}, for soil specimens tested in test series W3. As shown, the data yield the relationship from linear regression in the semi-log space

$$\log(k) = 9.813\ e_{ave} - 16.448 \quad [18]$$

with R^2 = 0.947. Again, the nature of the consolidation pressure either vacuum, surcharge, or a combination of both, does not show any effect to the hydraulic conductivity of the soil during the tests.

Reconstituted–consolidated soil sample

The soil specimens used for the reconstituted–consolidated soil tests, test series O1, were prepared from Orleans clay obtained from a site 2 km east of Orleans Township in Ontario. The soil properties are summarized in Table 2. The soil is classified as an inorganic clay of high plasticity with 64% clay and 35.9% silt. Chemical analysis and X-ray diffractions were performed to determine the mineral composition of the sediment.

The soil specimens were prepared using a specially designed consolidation column, 150 mm in diameter and 450 mm in height that is capable of consolidating clay slurry to a soil mass under a specific consolidation pressure. The slurry was mixed to a water content approximately two times the liquid limit (120%) and then gradually loaded in small load intervals over a period of 14 days until the soil was consolidated at 60 kPa. Four specimens were trimmed from the consolidated soil and tested as tests nos. O1.1, O1.2, O1.3, and O1.4 as summarized in Table 3. It is worth noting that the initial water contents of the four specimens vary slightly, as they were dependent on the specimen loca-

Fig. 6. Void ratio versus applied consolidation pressure for test series W3.

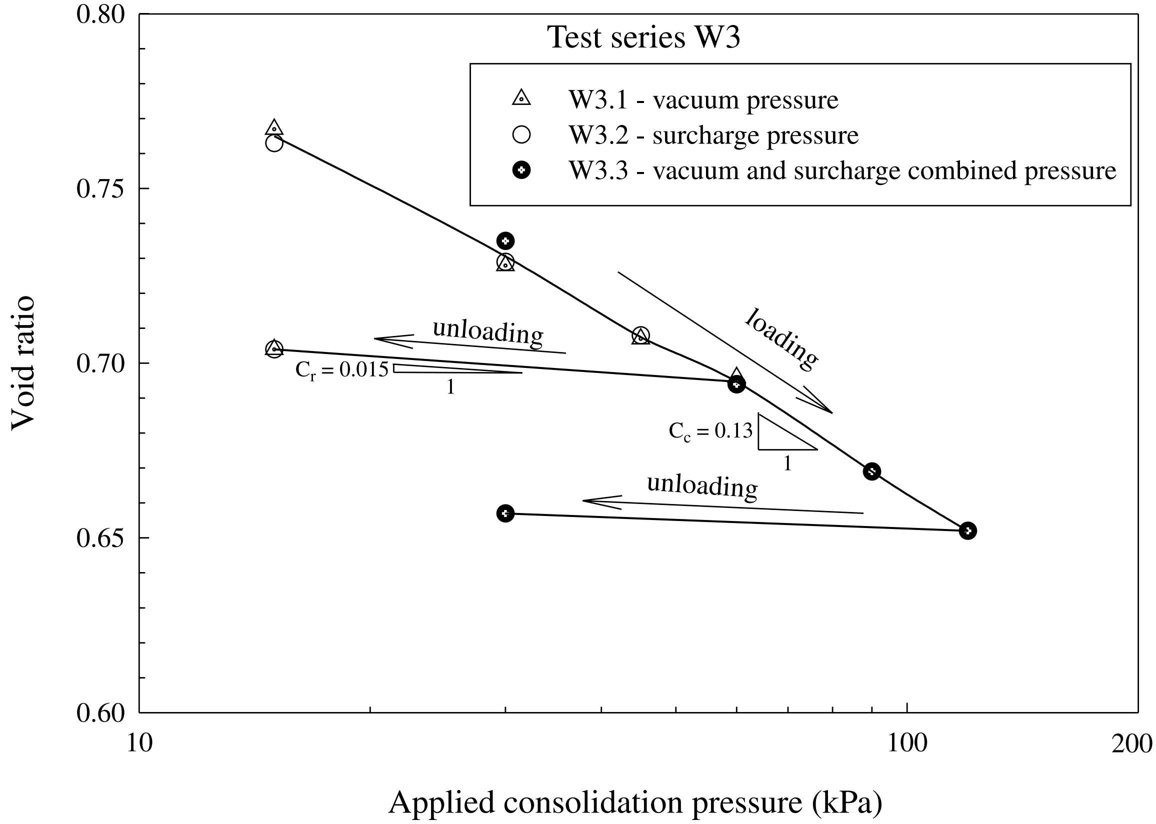


Fig. 7. Coefficient of consolidation versus applied consolidation pressure for test series W3.

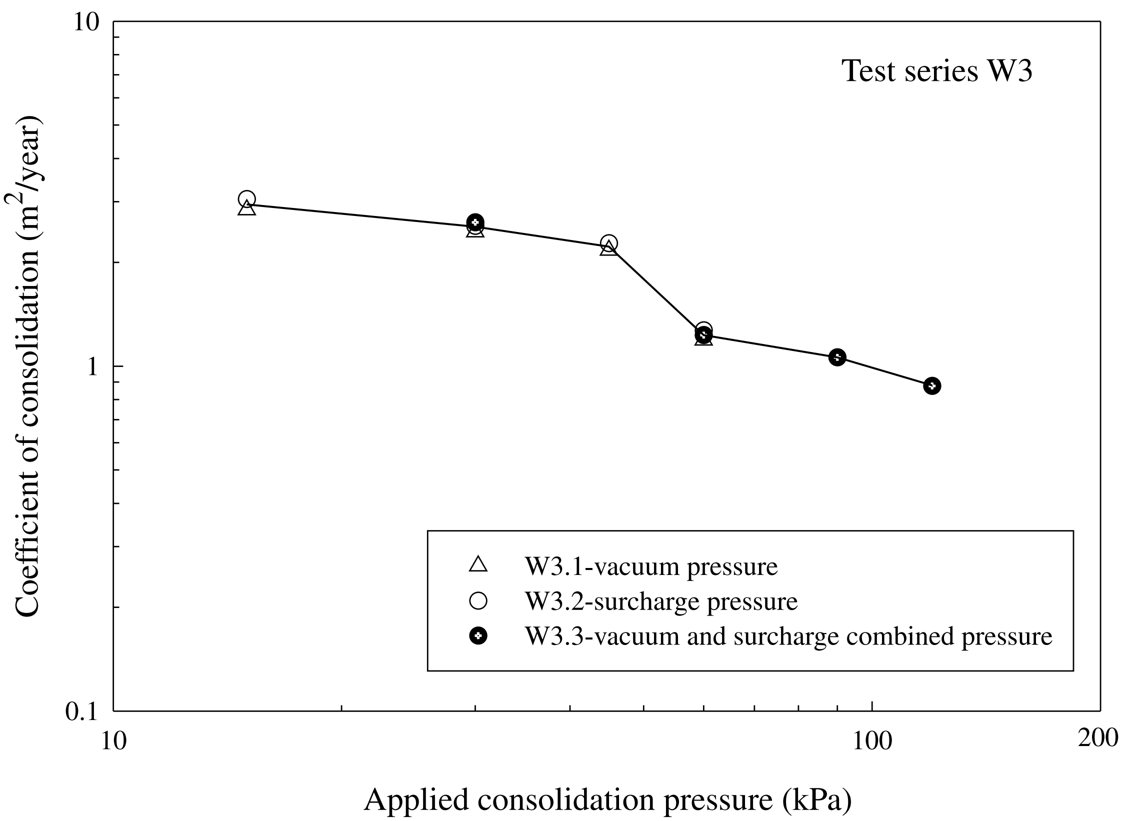


Fig. 8. Hydraulic conductivity versus average void ratio for test series W3.

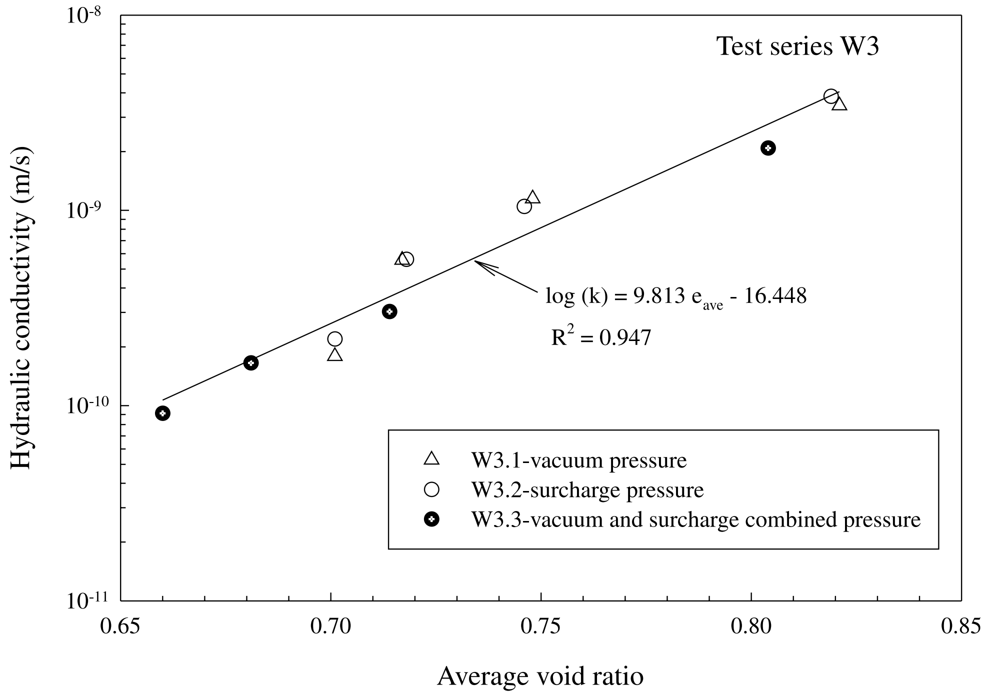


Table 2. Summary of properties of Orleans clay.

Soil properties	
Liquid limit, %	63–70
Plastic limit, %	31–39
Plasticity index, %	31–32
Specific gravity	2.80
Cation exchange capacity, mequiv./100 g of soil	16.3
Sand-sized, %	0.1
Silt-sized, %	35.9
Clay-sized, %	64
Mineralogy	
Chlorite, %	5–10
Illite, %	55–60
Quartz, %	14
Feldspar, %	7–10
Amphibole, %	5–6
Carbonate content, %	~1.1

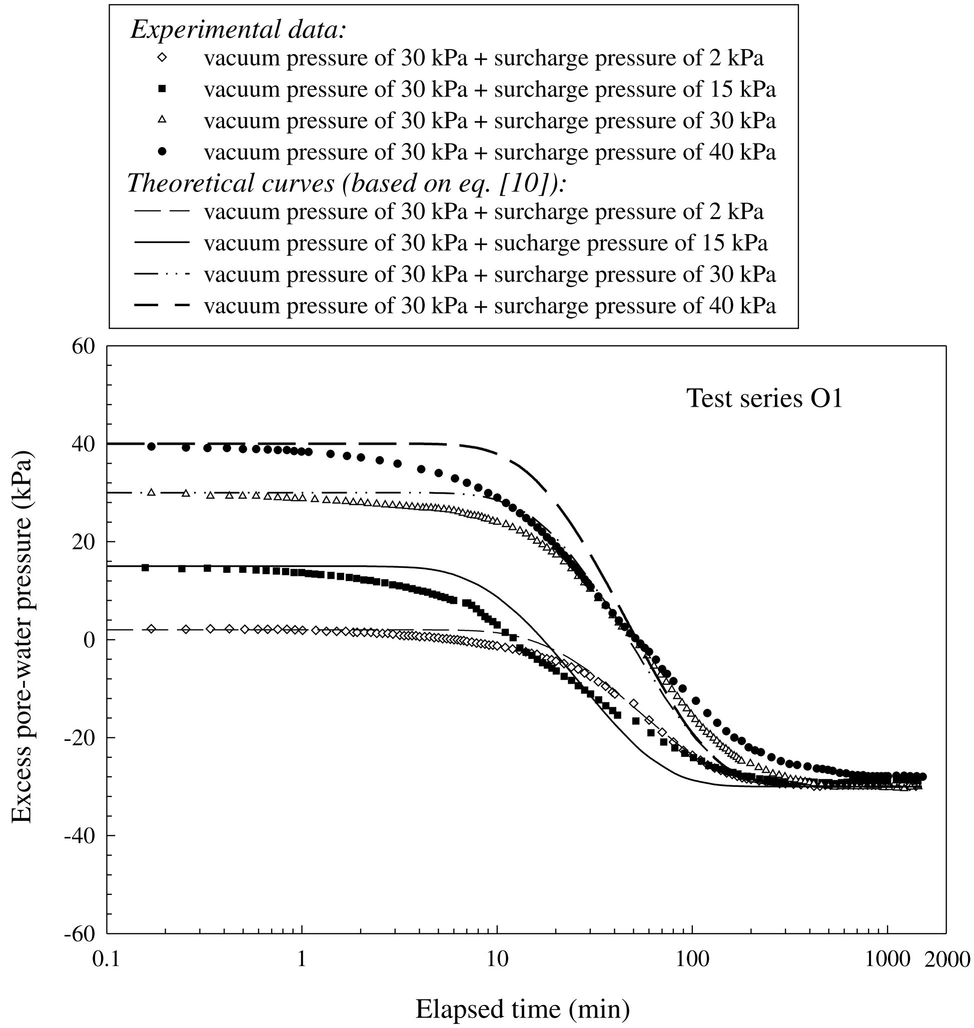
tion in the consolidation column. The first soil specimen, O1.1, was tested under a combined vacuum pressure of 30 kPa and surcharge pressure of 2 kPa; the second specimen, O1.2, was tested under a combined vacuum pressure of 30 kPa and surcharge pressure of 15 kPa; the third specimen, O1.3, was tested with a combined vacuum and surcharge pressure of 30 kPa each; and the fourth specimen, O1.4, was tested with a combined 30 kPa vacuum pressure and 40 kPa surcharge pressure. Since the soil specimens were reconstituted and consolidated at 60 kPa, they possess a preconsolidation pressure of the same magnitude. Therefore, the combined consolidation pressures applied on the soil specimens, ranging from 32 to 70 kPa, as shown in Table 3, are either lower than, equal, or higher than the preconsolidation pressure. Therefore, test series O1 represents vacuum and surcharge combined consolidation of either overconsolidated or normally consolidated clays.

After the primary consolidation was completed, as identified by the flattening of the excess pore-water pressure and settlement versus time curve, the test was terminated. Figure 9 shows the excess pore water versus the elapsed time of the tests. The figure also presents the corresponding excess pore-water pressures predicted from eq. [10]. As seen in the figure, the trends of the excess pore-water pressure in the four tests are in general agreement with the theoretical prediction. Figure 9 also shows that the pore-water pressures measured at the bottom of the soil samples converge to the applied vacuum pressure by the end of the test, indicating that vacuum preloading is equally effective under various surcharge pressures. The coefficients of consolidation were obtained for the four tests conducted from the best fit of the experimental results using Mathcad (MathSoft Inc. 2000) and are summarized in Table 3.

Discussion

The vacuum preloading has been applied extensively in land reclamation projects with or without concurrent application of surcharge preloading (Shang et al. 1998). The one-dimensional consolidation model (eq. [10]) and the results obtained using the Welland River sediment (test series W1, W2, and W3) provide the theoretical and experimental supports to the design of these projects, which were performed mainly based on empirical approaches. In particular, the results show that the vacuum pressure generates the same effect as that of a surcharge pressure under one-dimensional conditions. However, it should be cautioned that the lateral displacement of soil is completely different between the surcharge and vacuum preloading, as reported in case studies (Shang et al. 1998; Shang and Zhang 1999). Therefore further study on three-dimensional vacuum preloading consoli-

Fig. 9. Excess pore-water pressure versus elapsed time for test series O1.



dation is needed in terms of both theoretical and experimental aspects.

The fourth set of tests (test series O1) was conducted on Orleans clay soil specimens that have been reconstituted and consolidated to 60 kPa. This set of tests simulates the use of vacuum assisted consolidation for in situ strengthening of either normally consolidated or overconsolidated clays. The results of test series O1 indicate that vacuum preloading can be effectively used in combination with a surcharge load to consolidate a normally consolidated or overconsolidated soil.

Since the maximum vacuum pressure is limited by the vapour pressure of water, it cannot exceed 100 kPa in magnitude. In practice, a design value of 90 kPa is used (Shang et al. 1998). However, with combined vacuum and surcharge preloading, an equivalent consolidation pressure well above 90 kPa can be achieved.

It should be noted that the negative vacuum pressure generates suction in soil, which pulls the soil mass together and to some extent is analogous to the effect of capillarity in soils. Therefore, the designated maximum vacuum pressure may be applied immediately during operation without the risk of causing the shear failure in soil. This has long been recognized in field applications (Shang et al. 1998). Therefore, vacuum preloading is especially advantageous over surcharge preloading when the soil is very soft, such as in the case of hydraulic fill used in land reclamation projects.

Summary and conclusions

This research focuses on one-dimensional consolidation of clay soils induced by a vacuum pressure and a pressure generated by vacuum and surcharge loading. Terzaghi's one dimensional consolidation model is revisited with the initial and boundary conditions corresponding to vacuum and vacuum–surcharge combined loading. An experimental apparatus is designed and built to verify the assumptions made in the derivation of the model. The apparatus is capable of applying a designated vacuum pressure and a pressure generated by the combined vacuum and surcharge loading to a soil specimen. The pore pressure and settlement can be measured with real time during the consolidation process. Two natural clay soils recovered from Ontario, Canada were used to study the vacuum and vacuum–surcharge consolidation behaviour. The conclusions from the study can be summarized as follows:

Table 3. Summary results of the tests conducted.

Test No.	Applied vacuum pressure, p_v (kPa)	Applied surcharge pressure, q (kPa)	Initial void ratio, e_o	Final void ratio, e_f	Coefficient of consolidation, c_v (m^2/year)	Hydraulic conductivity, k (m/s)
Welland River sediment						
W1.1	20	0	0.874	0.747	2.24	2.44×10^{-9}
W1.2	0	20	0.874	0.743	2.30	2.58×10^{-9}
W1.3	20	20	0.874	0.700	2.44	1.84×10^{-9}
W2.1	30	0	1.092	0.821	3.68	5.29×10^{-9}
W2.2	0	30	1.092	0.824	3.59	5.09×10^{-9}
W2.3	30	30	1.092	0.763	4.10	3.63×10^{-9}
W3.1	15	0	0.874	0.767	2.84	3.44×10^{-9}
	30	0	0.767	0.728	2.45	1.15×10^{-9}
	45	0	0.728	0.707	2.17	5.56×10^{-10}
	60	0	0.707	0.696	1.19	1.7910^{-10}
	15	0	0.694	0.704	—	—
W3.2	0	15	0.874	0.763	3.05	3.84×10^{-9}
	0	30	0.763	0.729	2.55	1.05×10^{-9}
	0	45	0.729	0.708	2.27	5.61×10^{-10}
	0	60	0.708	0.694	1.27	2.19×10^{-10}
	0	15	0.694	0.704	—	—
W3.3	15	15	0.874	0.735	2.61	2.08×10^{-9}
	30	30	0.735	0.694	1.23	3.03×10^{-10}
	45	45	0.694	0.669	1.06	1.65×10^{-10}
	60	60	0.669	0.652	0.88	9.13×10^{-11}
	15	15	0.652	0.657	—	—
Orleans clay						
O1.1	30	2	1.823	1.716	3.44	1.29×10^{-9}
O1.2	30	15	1.742	1.625	3.99	1.20×10^{-9}
O1.3	30	30	1.736	1.604	2.63	6.76×10^{-10}
O1.4	30	40	1.837	1.555	2.79	1.30×10^{-9}

Note: —, indicates data not available. For test series W1, W2, and W3 the initial water contents were 32, 40, and 32%, respectively.

(1) The one-dimensional model of vacuum and vacuum–surcharge consolidation describes the consolidation behaviour of the soils well. Therefore, the model can be used in the design of soil improvement projects using vacuum andvacuum–surcharge preloading when the one-dimensional condition prevails.

(2) The vacuum pressure generates nearly identical effects compared to a surcharge pressure of the same magnitude under one-dimensional conditions, as evidenced by the responses of the soil pore pressure and settlement, as well as from the coefficient of consolidation and hydraulic conductivity. Hence, the soil compressibility characteristics obtained in conventional consolidation tests can be used in the design of vacuum consolidation.

(3) The vacuum preloading can be enhanced with surcharge preloading when the required consolidation pressure exceeds the limit of the vacuum pressure (in the magnitude of 90 kPa).

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